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STRUCTURAL DIVISION

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WIND LOADS ON GIRDER BRIDGES

John M. Biggs,[1] A. M. ASCE, Saul Namyet,[2] and Jiro Adachi[3]

INTRODUCTION

Wind loads have been considered in the design of bridges for many years but even now there is very little data available on which to base the computation of these loads. This lack of information probably exists for three reasons: (1) A completely satisfactory solution cannot be obtained analytically; (2) the facilities necessary for an experimental solution are not readily available to those most interested in the problem; and (3) for the majority of structures wind loads are of secondary importance. With regard to the last point it may be noted that in the case of the elevated highways which are becoming common in our large cities wind loads are of primary importance.

Of all the technical problems currently facing the structural engineer that of static wind loads is one which can be solved with relative ease. With a modest effort loading coefficients could be obtained for all common bridge types in a form easily applied to design. In the more important structures where unusual structural forms are encountered it is feasible to conduct wind tunnel tests for the purpose of determining the loads on that particular structure.

The experimental program described in this paper is a continuation of earlier work on truss bridges which was previously reported.[4] Presented herein are wind load data for several common types of railway and highway girder bridges. The railway types include single and double track bridges of both through and deck construction. The highway types include through and deck construction with two, three, and four parallel girders. Because of the relatively small number of structural forms used in practice, the railway types tested include the majority of railway bridges. However, there is a great variation in the form of highway bridges and it was possible in this program to include only the more common types. It is hoped that future work will expand the data in this respect.

All models were tested for various angles of attack (angle between the wind direction and the horizontal plane). Also included are data which indicate the effect of angle of yaw (wind direction in the horizontal plane). Wind directions in the vertical plane containing the longitudinal axis of the bridge are also considered since the resulting forces are important in the design of bascule bridges.

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4. "Wind Loads on Truss Bridges," John M. Biggs ASCE Proceedings - Separate No. 201, July, 1953.

Description of Models

Descriptions of the models are given in Tables I and II and photographs of a selected group are shown in Figs. 1 and 2. Five basic types of railway bridges and four types of highway bridges were included in the test program.

The wind load characteristics of a bridge depend not only upon the basic configuration of type but also upon the proportions of the cross section. The most important variable within a given configuration is the ratio of the girder depth to the other dimensions of the cross section. This parameter varies with the span of the bridge because the girder depth is, in general, a function of span. In order to take into account the aerodynamic effect of the girder depth - bridge width ratio each basic model type, with two exceptions, was tested with three different girder depths. Thus data for a total of twenty-four models were obtained.

The tracks, ties, floor system, and widths of the railway models were constructed to a scale of 1:32. The three girder depths used therefore correspond to 4, 8, and 12 feet at full scale. The scale of the highway models depends upon the width of the roadway. If the roadway is 24 feet wide the scale is approximately 1:25. At this scale the three girder depths correspond to approximately 3, 6, and 9 feet on the prototypes.

The models are constructed of wood and contain as much detail as is considered necessary to reproduce the aerodynamic characteristics of the prototypes (see Figs 1 and 2). In order to reduce the labor involved the girders and floor systems are constructed separately so that the same set of girders may be used in all models. The girders consist of three plates representing the web and flanges together with rectangular strips simulating stiffeners. In the railway models the track and ties are to scale and the floor beams and stringers of the floor system are made of three rectangular pieces also to scale. The roadways of the highway models are made of plywood slabs to which are glued rectangular strips representing stringers and floor beams. The bracing elements of all models are made of small rectangular strips.

The railings on the highway models are approximately to scale. The gaps in the railings which appear in the photographs are necessary to provide clearance for the mounting devices and have little effect on the total wind forces.

Test Procedures

All of the tests described herein were conducted in the M.I.T. wind tunnel facility. The tunnel used is of the circulatory, non-pressured type having a 4.5 foot by 6 foot rectangular test section. The tunnel and its instrumentation are of conventional design and need not be discussed here. The maximum speed possible with the rather large models of this investigation is 80 mph.

Wind tunnel tests on models are useless unless the force coefficients obtained are applicable to the prototype. In other words, for a given velocity the ratio of the force on the model to that on the prototype must equal the scale squared. The parameter normally used to prove this equality of the model and prototype is the Reynold's Number, R, which is defined as $\frac{VD}{v}$, where V is the wind velocity, v is the kinematic viscosity of air, and D is a representative dimension of the model or prototype. The minimum R for the tests of this investigation was 9.2×10^4 (computed for V = 80 mph, D = 1.5 ins., and $v = 1.6 \times 10^{-4}$ ft/sec^2). The maximum R which might be expected on a full scale girder bridge is about 11×10^6 (computed for V = 100 mph, and D = 12 feet).

A girder bridge is an assembly of sharp-edged objects and the Reynold's Number effect is essentially the same as that of these objects taken individually. For sharp-edged objects such as a flat plate presented normal to the wind, the drag coefficient is a constant for all Reynold's Numbers above a critical value of approximately 10^3 and within the range significant for bridges. Since the R's computed above for the model and prototype are both well above this critical value, it may be assumed that the force coefficients obtained for the model also apply to the prototype. In addition, the models were tested at velocities between 30 and 80 mph and the drag coefficient did not change within this range. This indicates that the models were being tested at a point above the critical Reynold's Number.

Another aerodynamic phenomenon which must be considered in wind tunnel testing is the effect of aspect ratio which is defined as the ratio of the length to the depth of the model. It is well known that drag coefficients vary with aspect ratio and in the case of flat plates the coefficient varies from approximately 1.2 to 2.0 as the aspect ratio increases from unity to infinity. For the case of actual girder bridges, the correct aspect ratio is not easily ascertained but a value of infinity must be assumed. This assumption is probably correct for deck structures since the abutments effectively seal off the end of the structure. The actual condition is less certain for through bridges but in most cases the depth-span ratio and the end conditions are probably such that the drag coefficient approaches that for an infinite aspect ratio. In any event the most severe case must be assumed for design purposes.

All of the models tested are considered to be sectional, i.e., they represent only a portion of the span. In order to achieve an infinite aspect ratio in the wind tunnel the flow of air around the ends of the model must be prevented. There are several methods by which this might be accomplished but in this investigation only two were found to be practical. The first of these involves the placement of two large baffle plates in the tunnel as shown in Fig. 3. The models were inserted between the plates with the minimum practical clearance on each side. Variations in angle of attack were obtained by rotating the model about the trunnion axis which is horizontal and perpendicular to the flow. The wind velocity was determined by means of Prandtl pressure tubes placed just forward of the opening between the baffle plates. These pressure measurements were calibrated and corrected for blockage effects as discussed below.

The second test method involves the use of dummy extensions to the models as shown in Fig. 4. The extensions are supported independently and only the forces on the active model are measured. Thus the end effects on the dummy extensions do not affect the data and an infinite aspect ratio is attained. This second method has the disadvantage that extensions must be constructed but it has the advantage that yaw and roll angles may be considered by rotating the model and dummies about a vertical axis. This motion is provided by the turntable in the floor of the tunnel upon which both the model and dummies are mounted.

When large, blunt objects such as bridge models are tested in a wind tunnel considerable error may be introduced as a result of the "blockage effect". This error results from the fact that the large model causes an excessive disturbance in the normal flow pattern at the test section. The magnitude and distribution of the velocities between the model and the tunnel walls are not the same as would exist in the corresponding scaled space around the prototype. Normally the velocity pressure ($\rho\frac{V^2}{2}$) measured at points on the tunnel

walls adjacent to the model is the free stream pressure but this is not true when there is a high degree of blockage.

To circumvent this difficulty the velocity pressure must be measured at a point well forward of the disturbance caused by the model and the measurements thus taken corrected according to the degree of blockage introduced by the model. The necessary correction factors were obtained by placing in the tunnel flat plates of various widths for which the correct drag coefficient was known. When the drag force on each plate was measured the effective velocity pressure for that plate could be computed using the known drag coefficient. The ratio of this effective velocity pressure to that given by the forward pressure tap is the blockage correction for that plate. Since it may be assumed that the blockage correction is a function of the total drag in the tunnel, the same correction applies to a model producing the same drag as the plate. By the use of several plate sizes a curve of blockage correction vs total drag was obtained and this curve provided the corrections for the tests on bridge models.

The same general procedure for determining the correction factors was used in both test methods described above. Several of the models were tested by both of these methods and the fact that the corrected values were in close agreement verifies the accuracy of the procedure followed.

Test Results

The test data is presented in the form of dimensionless coefficients in Figs. 6 through 15. The notation used is defined below.

Wind direction angles:

α = angle of attack = angle between the wind direction and the horizontal plane (see Fig. 5(a)), positive when the wind is acting upward on the bridge.

β = angle of roll = angle of wind direction in the vertical plane containing the longitudinal axis of the bridge (see Fig. 5(b)).

ψ = angle of yaw = angle of wind direction measured in the horizontal plane (see Fig. 5(c)).

Wind force components and coefficients:

$H = C_H A_H \rho \frac{V^2}{2}$ = horizontal force acting transversely on the bridge

$V = C_V A_V \rho \frac{V^2}{2}$ = vertical force on the bridge. Positive when upward.

$S = C_{SH} A_H \rho \frac{V^2}{2}$ or $C_{SV} A_V \rho \frac{V^2}{2}$ = longitudinal force on the bridge.

$M_a = C_M A_V b' \rho \frac{V^2}{2}$ = the moment of the wind forces about an arbitrary point "a" (see Fig. 5(a))

C_H, C_V, C_{SH}, C_{SV}, and C_M are dimensionless force coefficients.

A_H = total projected area as seen in elevation.

A_V = total projected area as seen in plan. In the case of railway bridges this includes track and ties as well as all structural elements seen in plan view.

$\rho \frac{V^2}{2} = 0.00256\ V^2$ lbs. per sq. ft. (V in mph)

b' = width of bridge (see Tables I and II)

The data for railway bridges when tested with varying angle of attack are given in Figs. 6 - 9 in terms of the coefficient C_H, C_V, and C_M. Similar data for highway bridges are given in Figs. 10 - 13. Each figure contains the data for all the models of a given type. The d/b ratios are listed and it is intended that for an actual structure this ratio be computed and the coefficients determined by interpolation.

The transverse horizontal and longitudinal force coefficients for the models when the angle of yaw is varied are given in Fig. 14. The transverse coefficient is plotted as a ratio to that at zero angle of yaw (C_{HO}) so that the curves indicate the effect of yaw rather than the basic differences between the models. Fig. 15 shows the vertical and longitudinal force coefficients as the angle of roll is varied. Only four of the models were tested in roll and yaw because these are considered to be representative of all models.

Discussion of Results - Railway Bridges

In order to compute the maximum wind loads for design purposes it is necessary to decide what maximum angle of attack could occur at the site of the structure. This value is difficult to determine because very little information is available on the subject and it depends, to some extent, on the topography at the site. As may be observed in Figs. 6 - 13 the loads are very sensitive to angle of attack and it is unfortunate that information is not available on this subject. For the purpose of this discussion it is assumed that the maximum possible angle of attack is plus or minus 15 degrees.

Transverse Forces

The maximum transverse pressure (H/A_H) within the probable range of angle of attack ($\pm 15^o$) is given in Table III for each of the railway bridge models tested. These pressures are based upon the elevation area, A_H, which does not include the additional area presented by the leeward girder or other elements not seen in elevation. They are computed for a wind velocity of 100 mph which is considered to be a reasonable value for design purposes. The A.R.E.A. Specification (1950) is 75 lbs. per sq. ft. (50 lbs. per sq. ft. on 1-1/2 times the elevation area) and it may be observed that in only one case (Model 3RTA) is the experimental pressure as large as this value. The minimum design pressure indicated (Model 5RDC) is only 63% of the maximum - a variation which is not considered by the specification.

Comparison of Models 1 and 3 reveals that the additional girder in Model 3 has an appreciable effect on the maximum pressure for models having the shallow girders (A) but not for those having the other two girder depths (B and C). The same conclusion is reached by a comparison of Models 4 and 5. This is probably due to the fact that when the girders are deep relative to girder spacing the windward girder creates a wake which completely envelopes the leeward girders thus preventing the development of drag forces on the latter.

It may be further observed that the effect of increasing girder depth relative to bridge width is not appreciable in the case of two - girder bridges. However, in the case of three- and four - girder bridges an increase in girder depth tends to decrease the horizontal pressure considerably. This may also be explained by the larger wake relative to bridge width produced by the deeper girders.

As a matter of interest, the models were also tested at angles of attack greater than those reported herein. It was found that the maximum horizontal force on the railway models generally occurred at an angle of attack of

approximately -40 degrees and that the magnitude of this force was about 10% greater than that recorded at -30 degrees.

Vertical Forces

Also given in Table III are the maximum upward pressures within the probable range of angle of attack. The maximum downward pressures are not considered significant for design purposes. In general, deck structures are subjected to larger vertical pressures than are through bridges. The maximum pressure measured (Model 5RDC) is 7.2 lbs. per sq. ft. which is 15% of the maximum horizontal pressure. These results indicate that in the design of open-decked railway bridges uplift wind pressures are not important.

Overturning Moments

The maximum moment about point "a" of each model (see Fig. 5) is recorded in Table III. In the absence of a specified vertical force the designer usually assumes that the overturning moment results from the horizontal force alone and that this force acts as the mid-depth of the structure. A value thus computed (max H x $\frac{h}{2}$) is given in Table III for the purpose of comparison with the experimental moment, M_a. The difference between these two quantities is due not so much to the presence of vertical pressures, which are small, but rather to a variation in the location of the resultant horizontal force. In the most severe case the experimental moment exceeds the computed value, max H x $\frac{h}{2}$, by 33 per cent (Model 5RDB). This is not serious inasmuch as overturning moments are not particularly important in this type of bridge.

Effect of Yaw

The results of the yaw tests are given in Fig. 14. For the two railway bridges tested in yaw the maximum horizontal force coefficient occurs at $\psi = 0$. It may therefore be concluded that the results obtained for railway bridges in the angle of attack tests (Figs. 6-9) may be used directly without further consideration of the yaw effect.

Also shown in Fig. 14 is the longitudinal force coefficient, C_{SH}. The maximum value of this coefficient for the railway models is 0.36 which indicates that the maximum longitudinal force is not more than 20 per cent of the maximum transverse force. It should be noted that this coefficient was obtained for models having dummy extensions on both ends (see Fig. 4) and should be considered to be applicable only to a center portion of the span. In some cases the effect of the exposed end of the structure might be appreciable.

Roll Tests

The force coefficients for wind directions in the vertical plane through the bridge axis are shown in Fig. 15. These orientations are impossible on fixed spans but are important in the design of bascule bridges.

Discussion of Results - Highway Bridges

Transverse Forces

The maximum transverse pressures (H/A_H) for a wind velocity of 100 mph are given in Table IV for the highway bridge models. These are the effective pressures on the elevation area which does not include elements which are hidden in elevation. In Table IV the experimental pressures are compared with those given by the A.A.S.H.O. Specification (1953) which requires that the effective area be taken as 1-1/2 times the elevation area plus 1/2 the area of all girders in excess of two. Thus the A.A.S.H.O. pressures given in Table IV are equal to 50 lbs. per sq. ft. times the ratio of this effective area to the elevation area.

The experimental values do not indicate a significantly large variation in pressure with model type. However, in general the deck structures are subject to higher pressures than are the through structures and the pressures tend to increase with girder depth. The specifications are apparently conservative. In no case is the experimental value (for 100 mph) greater than 60 per cent of the specified value. In one case (Model 9HDA) the experimental pressure is only 33 per cent of that given by the specification. Considering all models, the minimum wind velocity which would produce the specifications pressure is 129 mph. There is no general increase in transverse force as the number of girders is increased and the specifications seem to be erroneous in this respect.

No tests were run on highway bridges having open-type decks. However, other investigators[5] have shown that such structures are subject to greater transverse pressures than are those having closed decks.

Vertical Forces

The maximum upward pressures acting on the plan area are given in Table IV. It may be observed that these pressures are of considerable magnitude. In the most severe case (Model 9HDA) the vertical pressure is 58 per cent of the transverse pressure. Taking into account the differences in plan and elevation areas, this indicates that the total upward force is 3.9 times the total transverse force. For design purposes, however, the upward force is significant only insofar as it affects the overturning moment.

Overturning Moments

An inspection of the results for the highway bridges (Figs. 10-13) reveals a striking similarity between the C_V and C_M curves. This indicates that the overturning moment is due almost entirely to the presence of the vertical force and that the moment due to the transverse force is very small in comparison. Further investigation of the results reveals that the location of the resultant vertical force varies considerably and may lie anywhere within the windward half of the bridge width.

The maximum moment coefficients are given in Table IV. The magnitude of the moments indicated are not sufficient to cause overturning but may cause significant stresses in the two girders and their supports.

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5. "Aerodynamic Stability of Suspension Bridges," Part I, F. B. Farquharson, University of Washington Engineering Experiment Station, June, 1949.

Effect of Yaw

The results of the yaw tests shown in Fig. 14 indicate that for deck highway bridges the maximum transverse force occurs at zero angle of yaw (ψ). For through highway bridges the maximum occurs at $\psi = 20^{o}$ and is 9 per cent greater than that at $\psi = 0^{o}$. This increase is not appreciable and probably does not occur at the angle of attack at which the maximum transverse force has been computed in Table IV.[6] It may therefore be concluded that the transverse coefficients (C_H) for highway bridges given in Figs. 10-13 for the angle of attack tests may be used directly without modification for the effect of yaw.

The maximum longitudinal force coefficient (C_{SH}) for highway bridges shown in Fig. 14 is 0.41 which indicates that the maximum longitudinal force is not more than 30 per cent of the maximum transverse force. The A.A.S.H.O. specifies a value of 25 per cent.

Roll Tests

The wind load coefficients for wind directions in the vertical plane containing the longitudinal bridge axis are given in Fig. 15. The through highway bridge is subjected to smaller forces normal to the deck because of the effect of the projecting girder flanges.

CONCLUSIONS

Railway Bridges

(1) The type of structure has a considerable effect on the transverse wind pressure. The wind loads specified by the A.R.E.A. are sufficiently large for the most severe case but do not consider this variation with bridge type.
(2) Vertical forces and overturning moments are not significant for open-deck railway bridges.

Highway Bridges

(1) The A.A.S.H.O. Specification is conservative with respect to transverse forces in all cases considered herein. (2) The presence of longitudinal girders in excess of two does not appreciably increase the wind loads. (3) Vertical forces and overturning moments on solid-deck highway bridges may be a significant design consideration.

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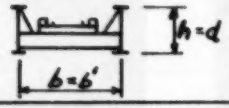
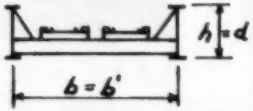
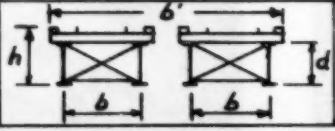
6. "Investigation of Wind Forces on Highway Bridges," G. S. Vincent, Highway Research Board, Special Report No. 10, 1953.

TABLE I SECTION DIMENSIONS OF RAILWAY BRIDGE MODELS

Model No.	Bridge Type		Elev. Area A_H ft^2	Plan Area A_V ft^2	h in.	b' in.	d in.	b in.
1RTA	Single Track Through Railway		0.312	0.681	1.50	5.50	1.50	5.50
1RTB			0.625	0.799	3.00	6.62	3.00	6.62
1RTC			0.937	0.847	4.50	6.62	4.50	6.62
2RTB	Double Track Through Railway (2 girders)		0.625	1.220	3.00	12.12	3.00	12.12
3RTA	Double Track Through Railway (3 girders)		0.312	1.285	1.50	11.00	1.50	5.50
3RTB			0.625	1.495	3.00	13.25	3.00	6.62
3RTC			0.937	1.565	4.50	13.25	4.50	6.62
4RDA	Single Track Deck Railway		0.375	0.509	2.00	2.81	1.50	2.81
4RDB			0.687	0.541	3.50	2.81	3.00	2.81
4RDC			1.104	0.570	5.00	2.81	4.50	2.81
5RDA	Double Track Deck Railway		0.375	1.02	2.00	8.81	1.50	2.81
5RDB			0.687	1.09	3.50	8.81	3.00	2.81
5RDC			1.104	1.15	5.00	8.81	4.50	2.81

All models 30 inches long. All ties 4.12 inches long.

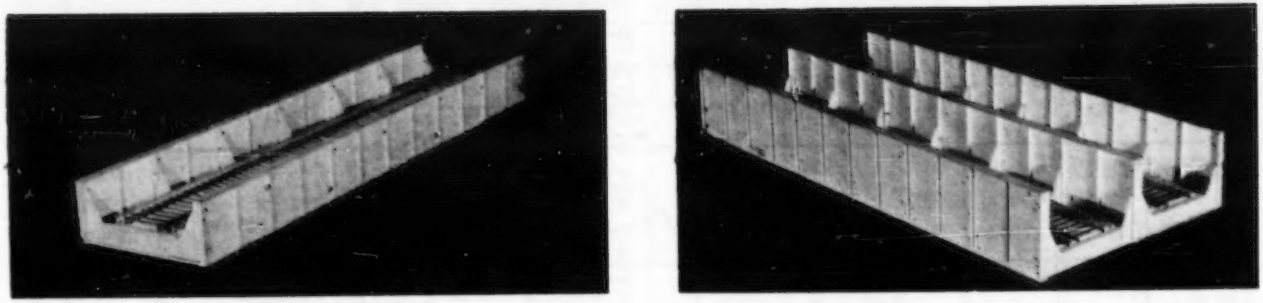


Figure 1 Typical Railway Models 1 RTB (left) and 3 RTC

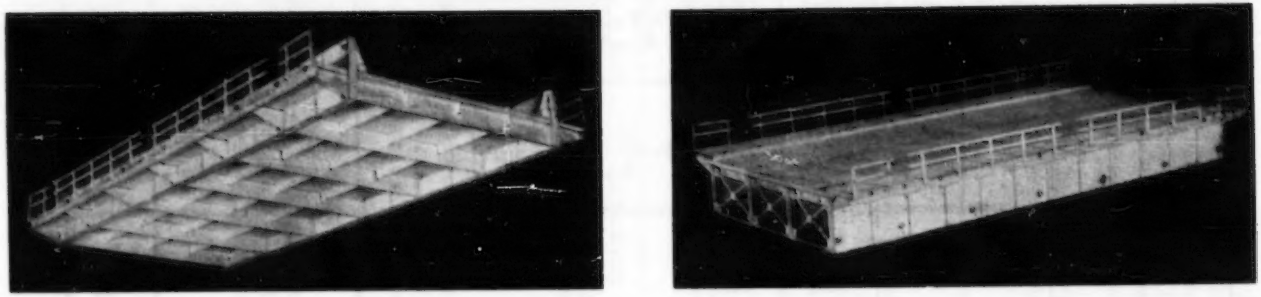
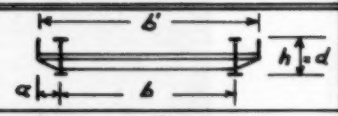
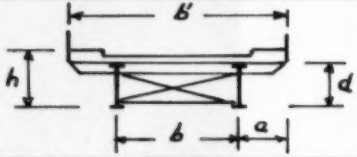
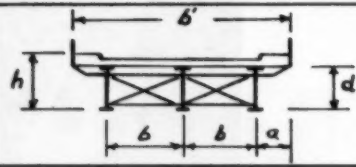
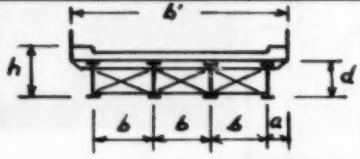


Figure 2 Typical Highway Models 6 HTB (left) and 9 HDB

TABLE II SECTION DIMENSIONS OF HIGHWAY BRIDGE MODELS

Model No.	Bridge Type		Elev. Area A_{H_2} ft^2	Plan Area A_{V_2} ft^2	h in.	b' in.	d in.	b in.	a in.
6HTB	Through		0.625	3.37	3.00	16.25	3.00	12.12	2.06
6HTC	Highway		1.065	3.37	4.50	16.25	4.50	12.12	2.06
7HDA	Two-Girder		0.499	3.37	2.12	16.25	1.50	8.00	4.12
7HDB	Deck Highway		0.813	3.37	3.62	16.25	3.00	8.00	4.12
7HDC			1.122	3.37	5.12	16.25	4.50	8.00	4.12
8HDA	Three-Girder		0.499	3.37	2.12	16.25	1.50	5.00	3.12
8HDB	Deck Highway		0.813	3.37	3.62	16.25	3.00	5.00	3.12
8HDC			1.122	3.37	5.12	16.25	4.50	5.00	3.12
9HDA	Four-Girder		0.499	3.37	2.12	16.25	1.50	4.00	2.12
9HDB	Deck Highway		0.813	3.37	3.62	16.25	3.00	4.00	2.12
9HDC			1.122	3.37	5.12	16.25	4.50	4.00	2.12

All Models 30 Inches Long

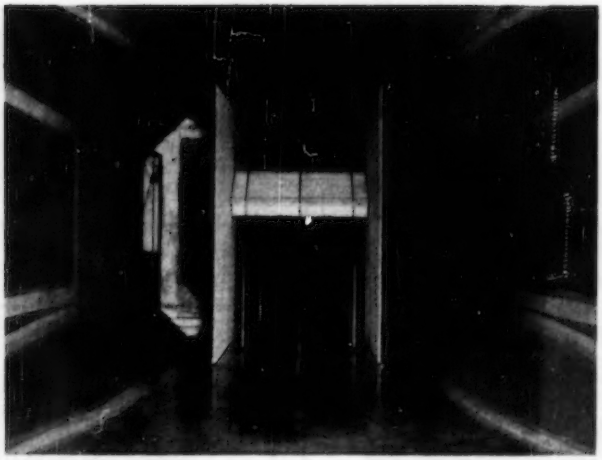


Figure 3 Test Method Using Baffle Plates

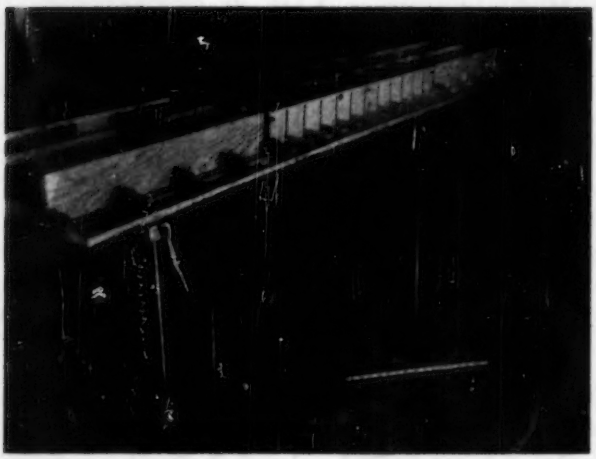
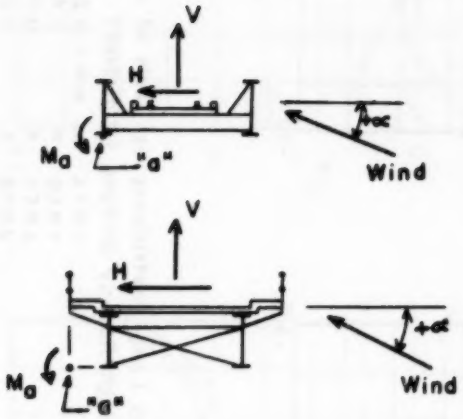
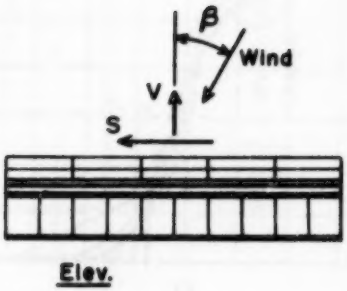


Figure 4 Test Method Using Dummy Extensions

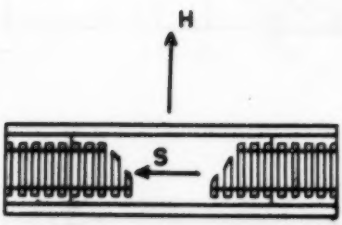


(a) Angle of Attack



Elev.

(b) Angle of Roll



Plan

(c) Angle of Yaw

FIG. 5. Wind Direction Angles and Force Components

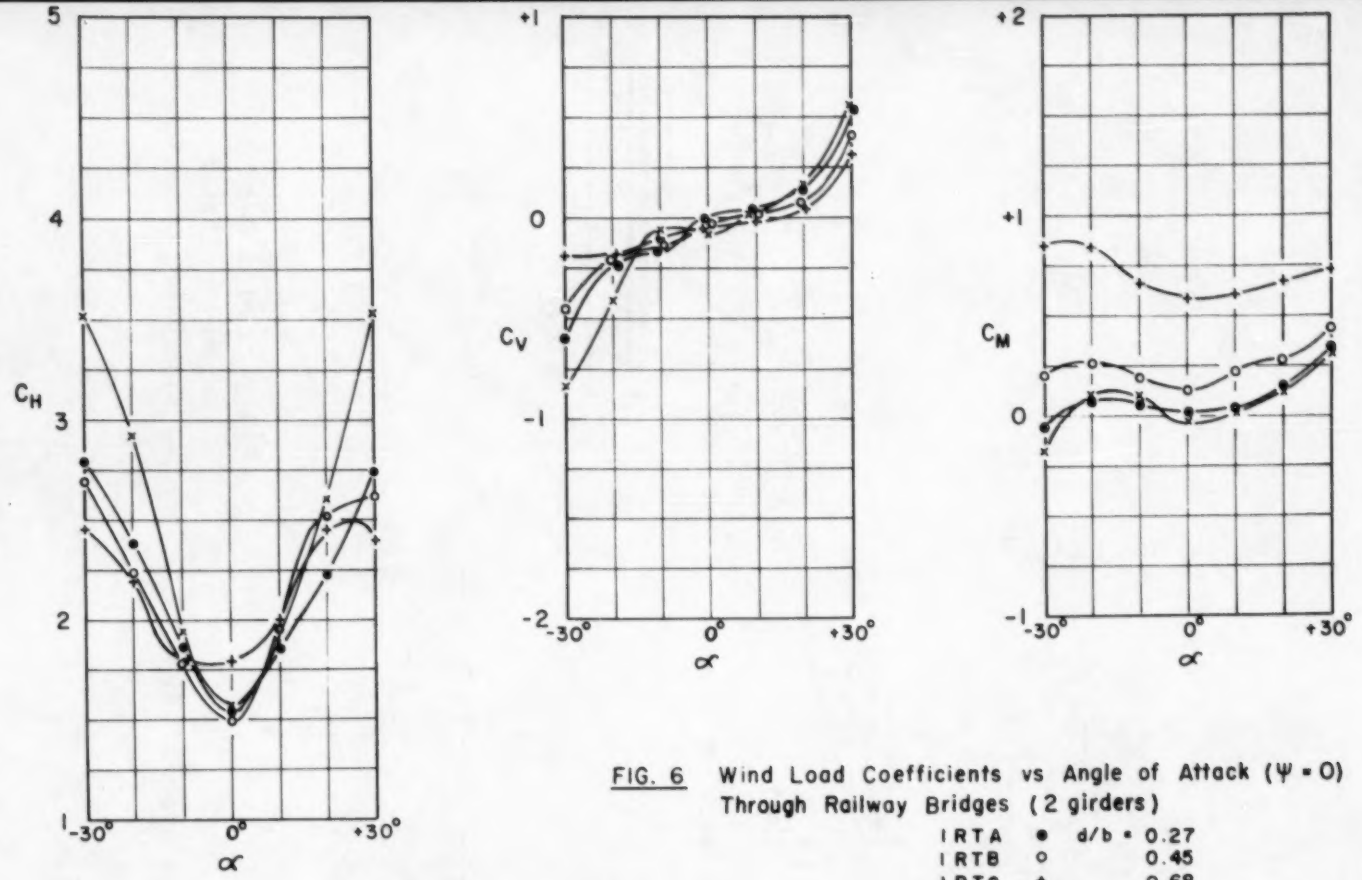


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FIG. 6 Wind Load Coefficients vs Angle of Attack ($\Psi = 0$) Through Railway Bridges (2 girders)

1RTA	●	d/b = 0.27
1RTB	○	0.45
1RTC	+	0.68
2RTB	×	0.25

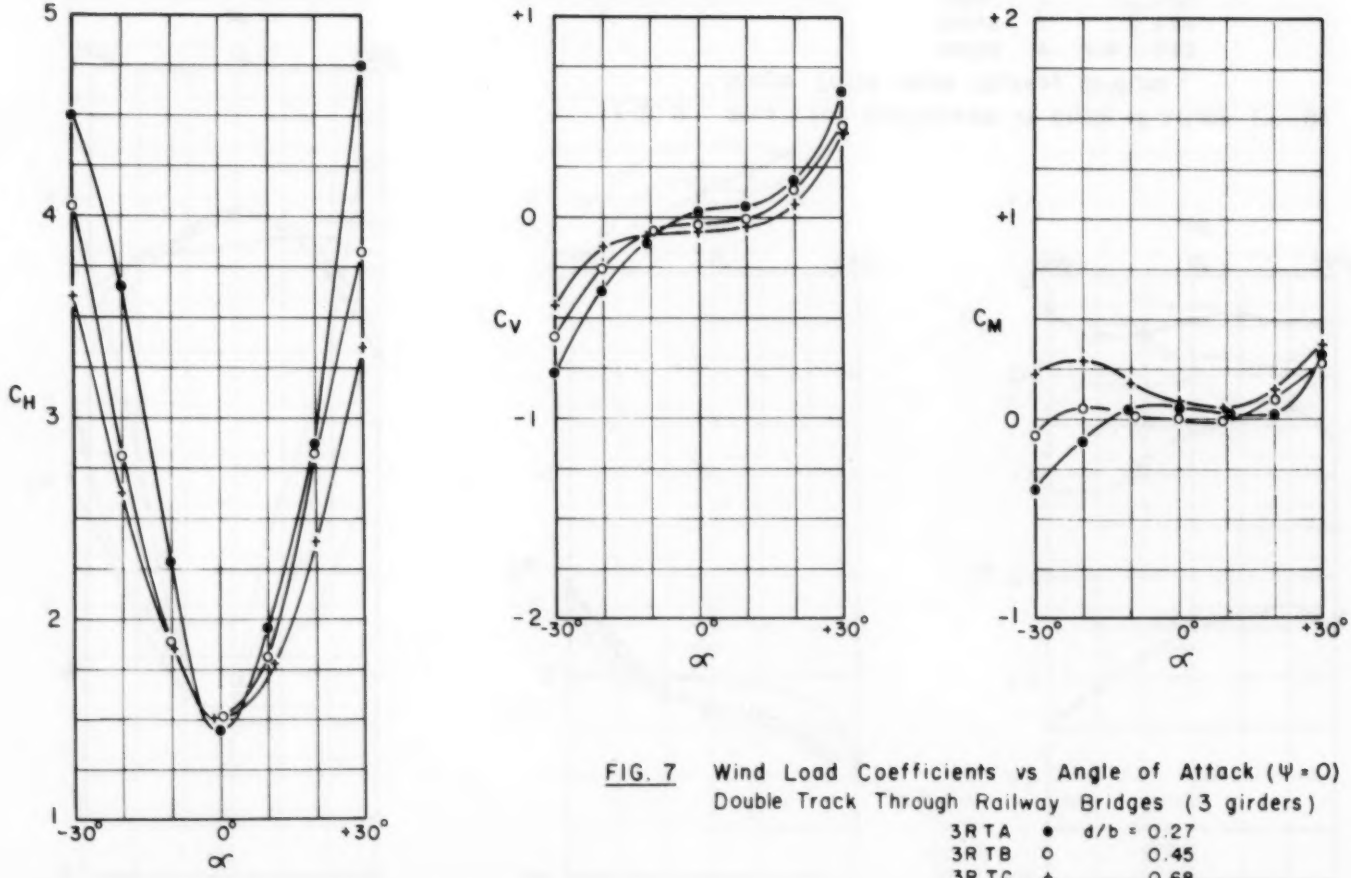
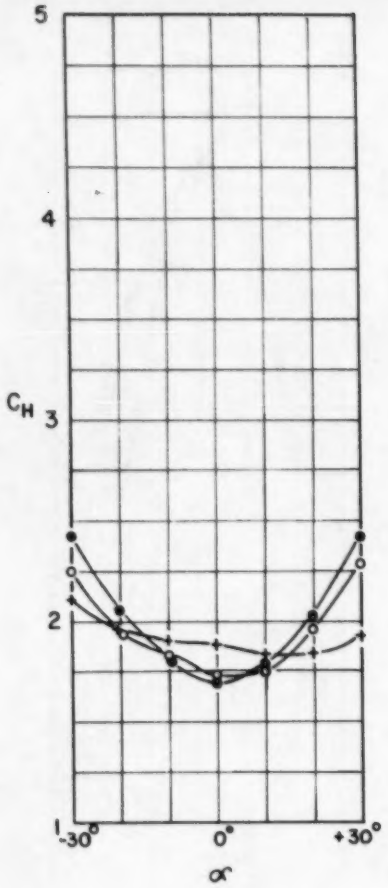
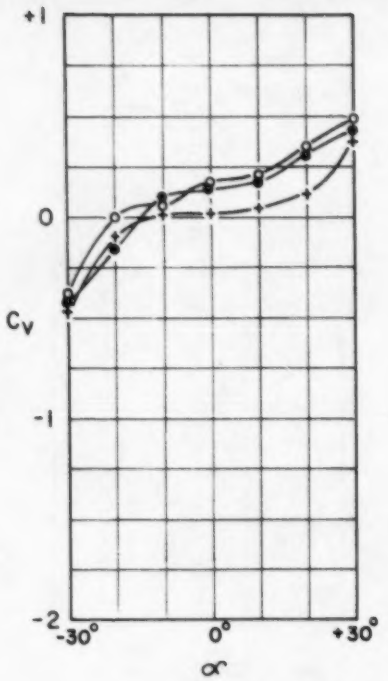
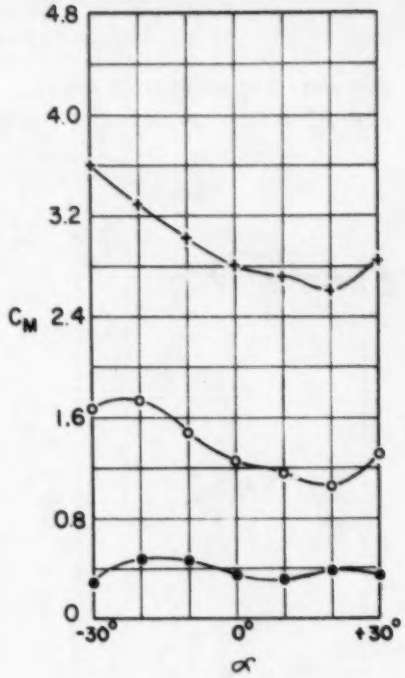


FIG. 7 Wind Load Coefficients vs Angle of Attack (Ψ = 0)
Double Track Through Railway Bridges (3 girders)

3R TA	●	d/b = 0.27
3R TB	○	0.45
3R TC	+	0.68







587-16

FIG. 8 Wind Load Coefficients vs Angle of Attack ($\Psi = 0$)
Single Track Deck Railway Bridges

4RDA	●	d/b = 0.53
4RDB	○	1.07
4RDC	+	1.60

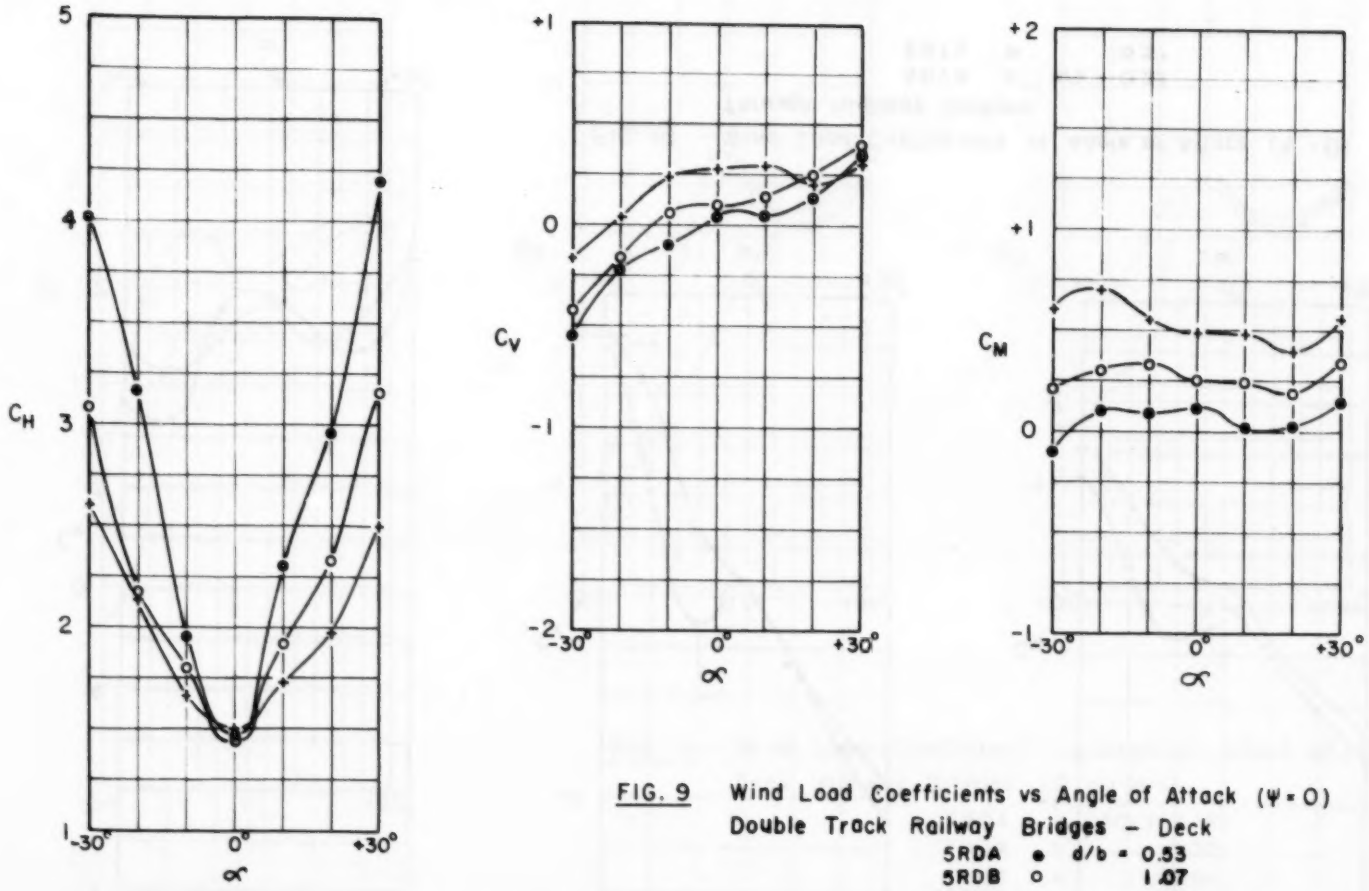
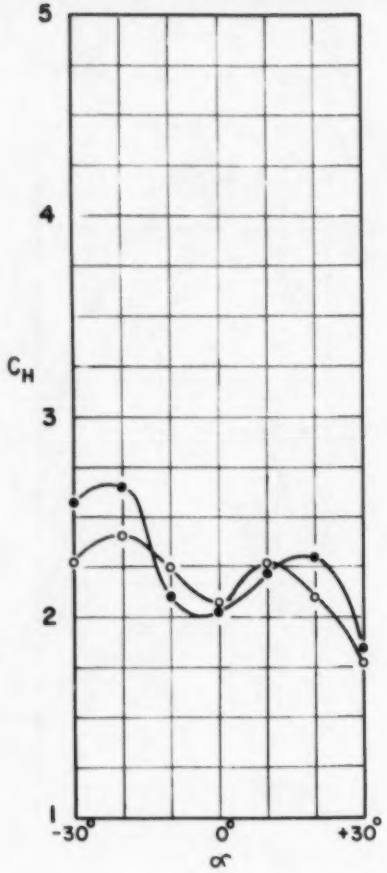
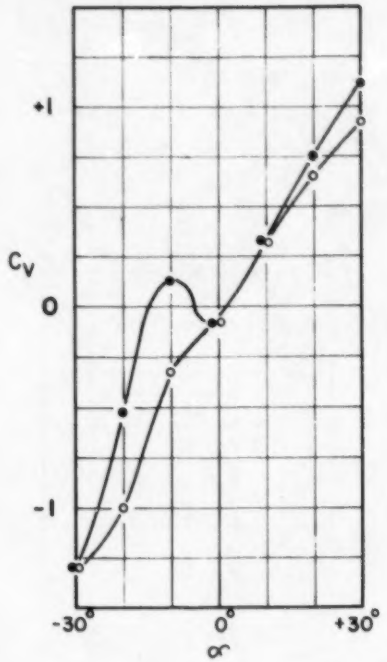


FIG. 9 Wind Load Coefficients vs Angle of Attack ($\Psi = 0$)
Double Track Railway Bridges – Deck

5RDA	●	d/b = 0.53
5RDB	○	1.07
5RDC	+	1.60





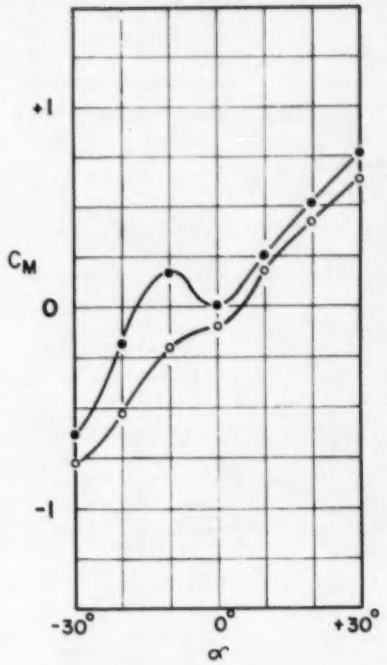


FIG. 10 Wind Load Coefficients vs Angle of Attack ($\Psi = 0$) Through Highway Bridges

6HTB ○ d/b = 0.25
6HTC ● 0.37

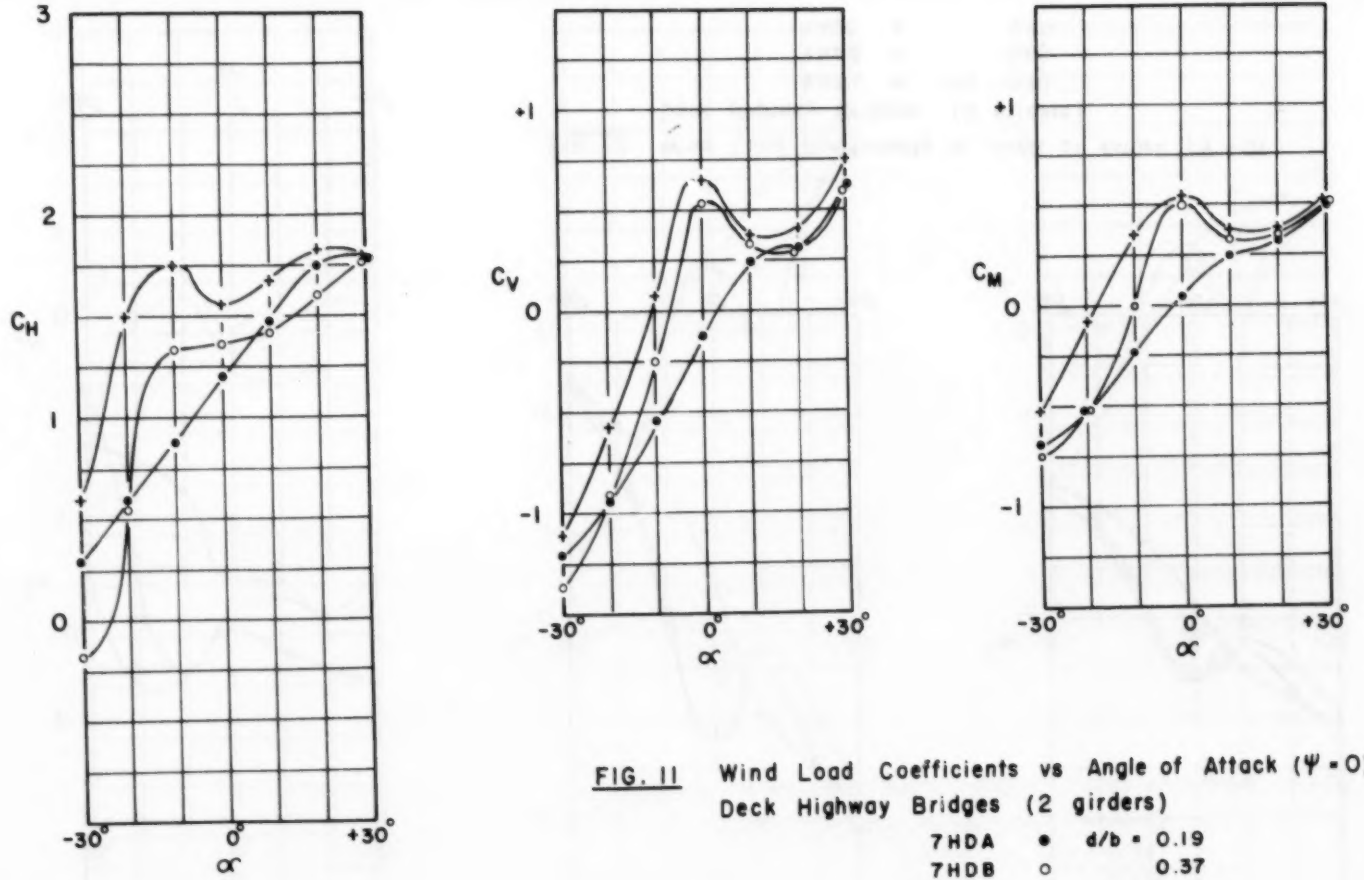


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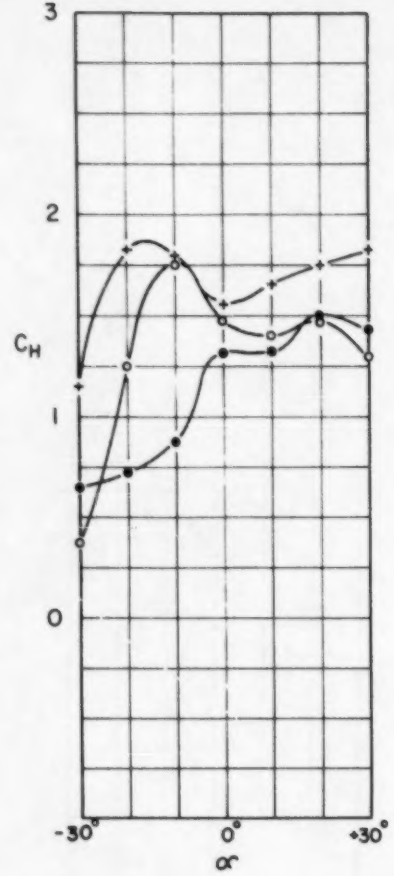
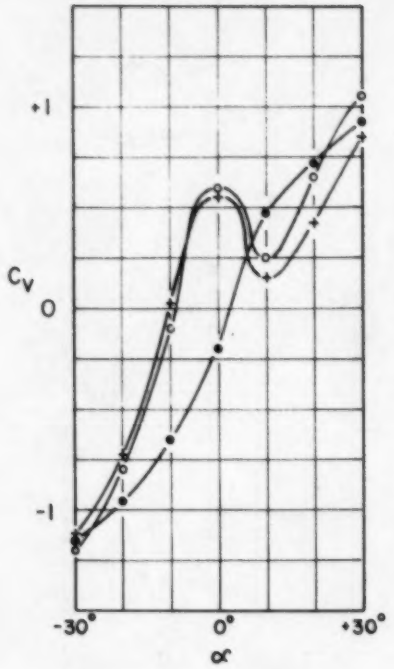
FIG. 11 Wind Load Coefficients vs Angle of Attack ($\Psi = 0$)
Deck Highway Bridges (2 girders)

7HDA	•	d/b = 0.19
7HDB	○	0.37
7HDC	+	0.56





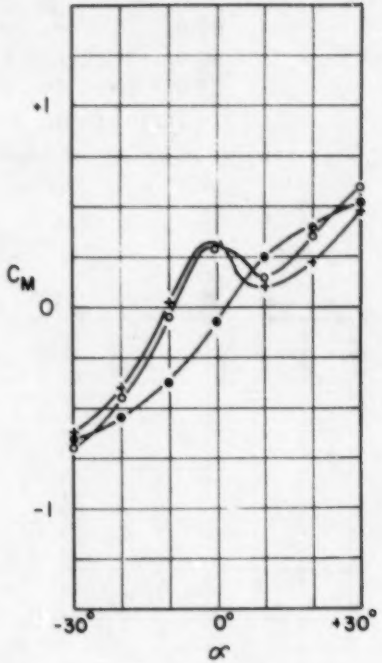
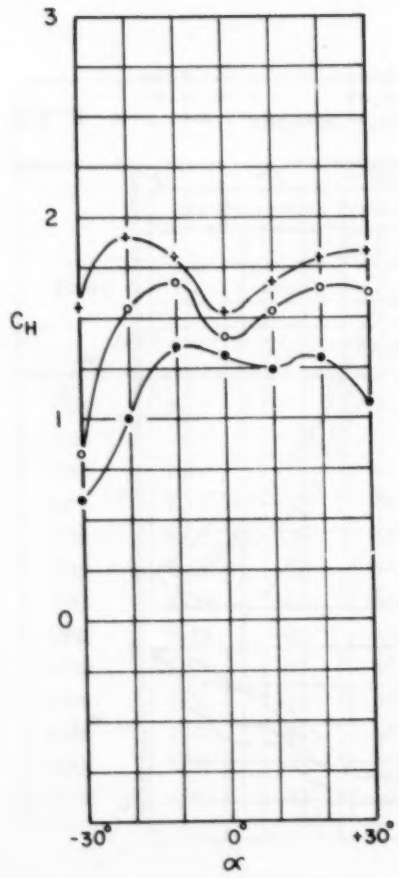
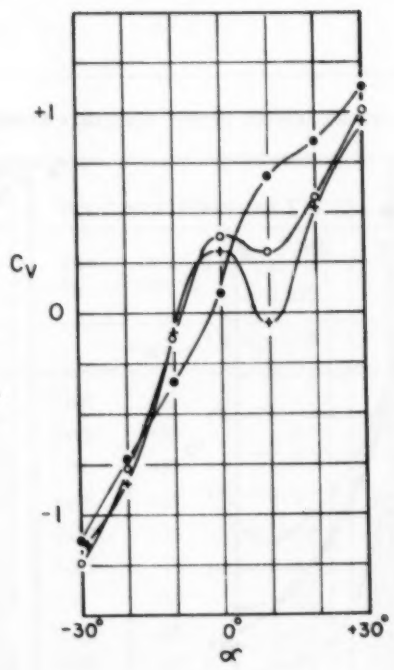


FIG. 12 Wind Load Coefficients vs Angle of Attack ($\Psi = 0$)
Deck Highway Bridges (3 girders)

8HDA	●	d/b = 0.30
8HDB	○	0.60
8HDC	+	0.90





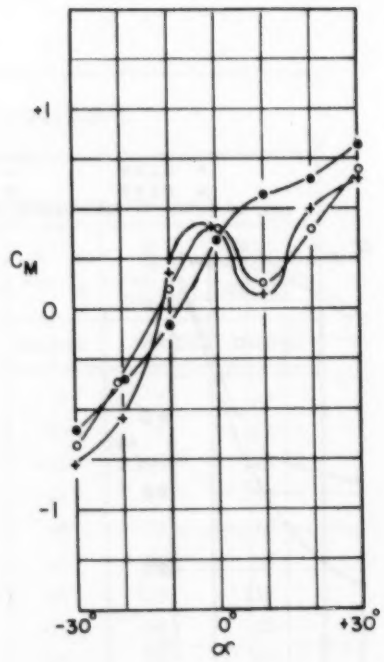


FIG. 13 Wind Load Coefficients vs Angle of Attack ($\Psi = 0$)
Deck Highway Bridges (4 girders)

9HDA	●	d/b = 0.37
9HDB	○	0.75
9HDC	+	1.12

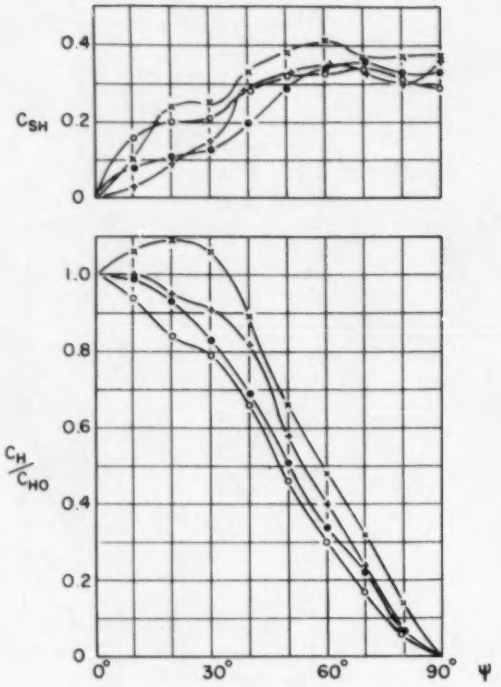
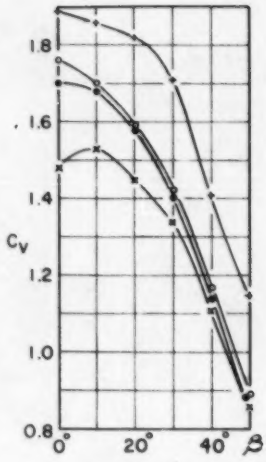


FIG 14 Wind Load Coefficients vs Angle of Yaw ($\alpha = 0$)

1RTB	●	6HTB	×
4RDB	○	8HDB	+



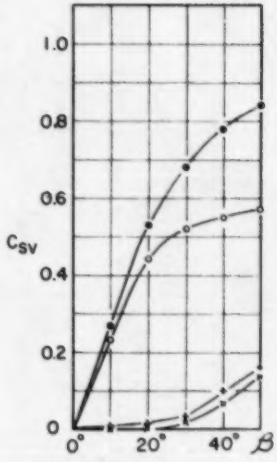


FIG. 15 Wind Load Coefficients vs Angle of Roll

1RTB	●	6HTB	×
4RTB	○	8HTB	+

TABLE III RAILWAY BRIDGES - MAXIMUM FORCES WITHIN POSSIBLE RANGE OF α ($\pm$ 15°)

Model	Max Transverse Pressures				Max Upward Pressures				Max Overturning Moment, M_a			
	Max C_H	α for Max C_H	Max H/A_H at 100 MPH		Max C_V	α for Max C_V	Max V/A_V at 100 MPH		Max C_M	α for Max C_M	M_a at 100 MPH	Max $Hx\frac{h}{2}$ at 100 MPH
		degrees	lbs/ft^2			degrees	lbs/ft^2			degrees	lb-ins.	lb-ins.
1RTA	2.12	-15	54.2	A.R.E.A. Specification - 75 lbs/ft^2	0.08	+15	2.0	No A.R.E.A. Specification	0.09	+15	9	13
1RTB	2.37	+15	60.6		0.04	+15	1.0		0.27	+15	37	57
1RTC	2.28	+15	58.4		0	+15	0		0.75	-15	108	123
2RTB	2.45	-15	62.7		0.08	+15	2.0		0.13	-15	49	59
3RTA	2.93	-15	75.0		0.10	+15	2.6		0.06	-5	22	18
3RTB	2.30	-15	58.9		0.05	+15	1.3		0.05	-15	25	55
3RTC	2.19	-15	56.0		0	+15	0		0.27	-15	113	118
4RDA	1.93	-15	49.4		0.23	+15	5.9		0.49	-15	18	18
4RDB	1.88	-15	48.1		0.27	+15	6.9		1.64	-15	64	58
4RDC	1.93	-15	49.4		0.07	+15	1.8		3.14	-15	129	136
5RDA	2.67	+15	68.3		0.08	+15	2.0		0.12	0	26	26
5RDB	1.97	+15	50.5		0.21	+15	5.4		0.33	-10	81	61
5RDC	1.86	-15	47.6		0.28	+10°	7.2		0.65	-15	169	132

TABLE IV HIGHWAY BRIDGES - MAXIMUM FORCES WITHIN POSSIBLE RANGE OF α ($\pm$ 15°)

Model	Max Transverse Pressures				Max Upward Pressures				Max Moment	
	Max C_H	α for Max C_H	Max H/A_H at 100 MPH	AASHO Spec.	Max C_V	α for Max C_V	Max V/A_V at 100 MPH		Max $C_M(+)$	α for Max C_M
		degrees	lbs/ft^2	lbs/ft^2		degrees	lbs/ft^2			degrees
6HTB	1.42	-15	36.3	75.0	0.50	+15	12.8	No A.A.S.H.O Specification	0.33	+15
6HTC	1.36	-15	34.8	75.0	0.56	+15	14.3		0.41	+15
7HDA	1.65	+15	42.2	75.0	0.31	+15	7.9		0.28	+15
7HDB	1.50	+15	38.4	75.0	0.53	0	13.6		0.50	0
7HDC	1.76	+15	45.0	75.0	0.65	0	16.7		0.55	0
8HDA	1.42	+15	36.3	90.5	0.61	+15	15.6		0.34	+15
8HDB	1.75	-10	44.8	94.1	0.60	0	15.4		0.29	0
8HDC	1.87	-15	47.9	95.8	0.56	0	14.3		0.32	0
9HDA	1.36	-10	34.8	106.1	0.78	+15	20.0		0.61	+15
9HDB	1.68	-10	43.0	113.3	0.38	0	9.7		0.40	0
9HDC	1.87	-15	47.9	116.7	0.31	0	7.9		0.43	-5

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c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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